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Failure of a Salt Aragonite Storage Pad

Yong S. Chae
Rutgers University, Piscataway, New Jersey

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Yong S. Chae
Professor of Civil Engineering and Associate Dean, Rutgers University, Piscataway, N.J.

SYNOPSIS A case study of the performance and failure of an aragonite and salt storage pad covering an area of approximately 250 ft. (76 m) square built on marginal subsurface conditions is reported. A detailed analysis of the failure illustrates the importance of thorough exploration of subsurface conditions and field instrumentation to monitor the performance of such storage pads during and after the loading period.

INTRODUCTION

Because of the scarcity of available land, aragonite and salt are often stored in areas where subsurface conditions are marginal. The reported storage pad was built on such an area where the subsurface soil consisted mostly of organic silt with high groundwater table. The failed salt storage pad is located north of the existing aragonite and salt storage area, as ss shown in Fig. 1, and covers an area of approximately 250 foot (76 m) square. The pad is bounded on the north by a swamp and marsh area with a creek dividing the pad and the marsh area, and on the south by the aragonite and storage pad.

After the stockpile height reached 28 feet (8.5 m) in 7 days of loading and remained at that height for another 16 days, a catastrophic failure occurred in a typical composite mode of translation and rotation in shear. Although damage could not be fully ascertained until the salt was removed from the pad, it nevertheless appeared that there had been a series of rotational failures to the north. The pad itself had moved some 12 feet (3.7 m) to the north, while the ground underneath had erupted onto the boundary by the creek. To the uninitiated, the damage looked similar to that caused by an earthquake.

GEOLOGY AND SUBSURFACE SOILS

Geologically the site is located in the Glaciated Piedmont area of New Jersey. The Engineering Soil Survey Map of New Jersey designates the deposits of the area as F/MTM, meaning "fill" and "marine tidal marsh".

Fig. 1 shows the general site and the location of boring and test pits performed at the site by the design firm. It should be noted that only one test boring was taken at the northeastern corner of the site, ironically in the area of failure, and six test pits were made. Based on information provided by the design firm's Soil Investigation Report, the subsurface condition at the site is shown in Fig. 2. Two organic silt deposits are present throughout the site. The upper layer is about 12 ft

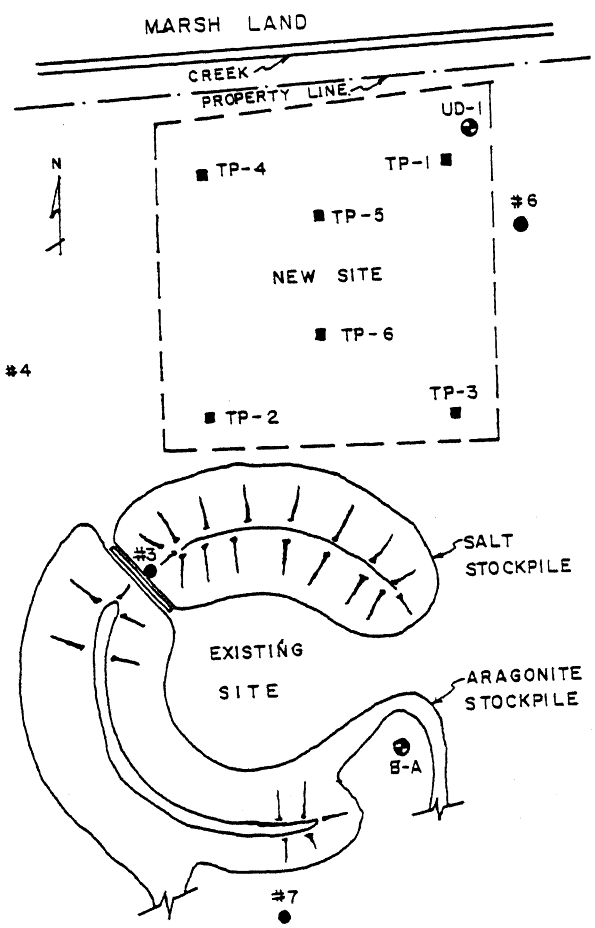

Fig. 1 General Site and Boring Locations

(3.7 m) thick and the lower layer about 28 ft (8.5 m) thick. These organic silt layers are separated by a sand-gravel stratum approximately 6 ft (1.8 m) thick, and the lower organic deposit is underlain by sands and gravels. The groundwater table is located approximately 8 ft (2.4 m) below the ground surface.

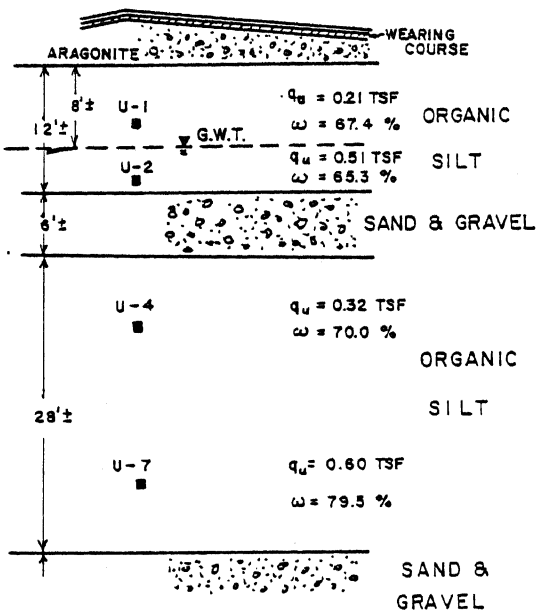

Fig. 2 Soil Profile and Properties

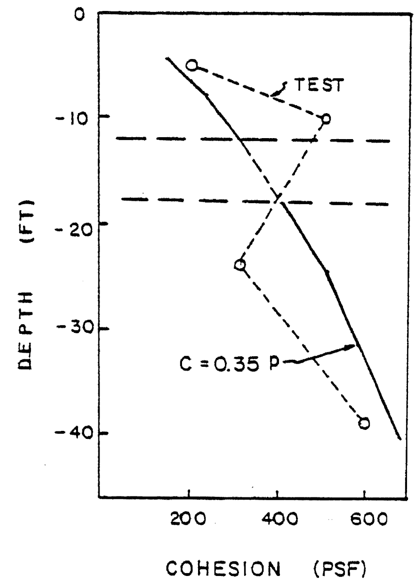

Fig. 3 Shear Strength of Organic Soil

Properties of Organic Silt

Coastal marshland soils at the site are typically of organic soils. The low specific gravity of both organic matter and water leads to low unit weights and, in general, low shear strength. The laboratory test results reported by the design firm give the shear strength in terms of cohesion (c) to be from 0.10 to 0.30 tsf (9.6 to 28.7 kN/m^2) and it generally increases with the pressure (p) under which the soil has consolidated. The ratio (c/p) is found(Tschebotarioff, 1973) to increase from a value of 0.18 for the liquid limit W_L = 40% to a value of 0.38 for W_L = 110%. For a value of W_L = 100%, which is approximately representative of the soil at the site, one can therefore approximate a cohesion value to be

$$c = 0.35\ p \qquad (1)$$

The correlation between Eq. (1) and the actual test results is shown in Fig. 3. It is seen that there is a reasonable trend of cohesion increasing with the consolidation pressure. A very high cohesion value at a shallow depth of 10 ft (3.1 m) may be due to overconsolidation from the dredged material.

Consolidation tests were run by the design firm n four specimens. Unit-strain and log pres- ıre curves showed inconsistent results in ɜrms of compression index and preconsolidation load. Only two compression-time curves were provided, one each for the upper and lower stratum, to this investigator. It is noted that between these two specimens there was almost a fivefold difference in the value of coefficient of consolidation. These are contradictory, because the soil represented by the first specimen is supposed to have consolidated to a greater degree due to surcharge from dredging.

ORIGINAL DESIGN

The original design was made based on the assumption of a stockpile configuration approximately 30 ft (9.1 m) high covering an area about 230 ft (70 m) square and having 45° side slopes. Their stability analyses indicated the stockpile to be safe against translational failure with a factor of safety of about 1.3. Settlement computations based on the laboratory consolidation test data resulted in predicted maximum primary settlements in the range of 3 to 4 feet. Under full load, about one half of the total settlement was expected to occur in about 2 years, with the remaining settlement taking an additional 10 years. Such settlements were not anticipated to present any serious problems with respect to the stored product.

Based on the shear strength data obtained in the field and laboratory for the upper organic layer, the designer proposed constructing a blanket of compacted granular material, and placing pavement on the granular blanket. Their evaluation indicated that to adequately dissipate heavy wheel loads from trucks and/or loading equipment, a granular blanket 2.5 ft (76 cm) thick was necessary. An asphalt pavement consisting of 4 in (10 cm) of stabilized base covered with a wearing course of 2 in (5 cm) FABC was constructed to provide a suitable storage pad. In order for the surface of the pad to drain and to compensate for the differential settlement between the central and the outer portions of the pad, a 2 ft (61 cm) crown was also provided as shown in Fig. 2.

LOADING AND FAILURE

Fig. 4 shows the stress (loading) history at the site. It is seen that the pad was loaded to 28,600 tons (2.6×10^5 kN) in 7.5 days, reaching a height of 28 feet (8.5 m). The failure occurred approximately 15 days after the completion of loading to the maximum height, or approximately 23 days after the inception of loading.

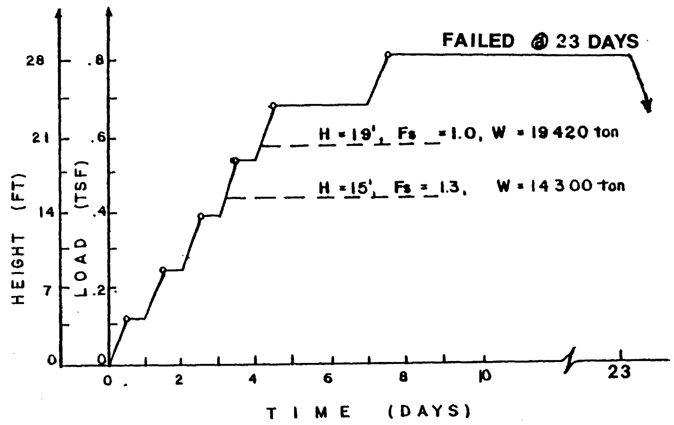

Fig. 4 Loading History

The exact geometrical configuration at the time of loading completion is not known to this investigator. However, assuming a trapezoidal piling with a one-on-one slope, the loading of 28,600 tons (2.6×10^5 kN) and height of 28 feet (8.5 m) represent a loaded area at the bottom of approximately 35,340 sq. ft (3284 sq. m) or 188 ft (157.3 m) square. This was approximately two-thirds of the originally proposed pad of 230 ft (70 m) square resulting in an increase of loading intensity to 0.81 tsf (77.6 kN/m^2) from the original design of 0.54 tsf (51.7 kN/m^2).

STABILITY ANALYSIS

An essential requirement when considering loading of a compressible material, such as organic silt, is to make detailed subsoil investigation, laboratory tests and design analysis, and to provide high quality field monitoring and inspection. None of these were done by the designer for this particular project. The controlling factor in the stability analysis is the shear strength of the soft organic stratum. Such deposits have invariably low shear strength, and this means that abrupt changes in height of fill placed over them must be limited to avoid shear failures in the form of large slides. It is not uncommon for the first layer of fill placed over marsh deposits in heights of 2 to 3 feet (61 to 91 cm) above water level to cause slides at the edges of advancing fill. Thus, thickness of fill layers and placement methods must be controlled strictly to avoid slides resulting from incompatibility between the shear strength of the soil and the loading intensity.

Stability Analysis Without Considering Gain in Shear Strength from Consolidation

The mechanics of slope stability is illustrated in Fig. 5. When the height of fill becomes excessive, either a translational, rotational or a combined mode failure will occur. Fig. 5 depicts a translation-rotational failure which probably was the case at this site. Because of the sand-gravel layer between the two organic silt strata, the shear failure plane is more likely along the A-B-C-D surface, precluding a deeper slide. In the analysis of failure of this nature, horizontal pressure is a very significant factor in consideration. In the case of soft soil loading, horizontal displacement occurs, as was the case here.

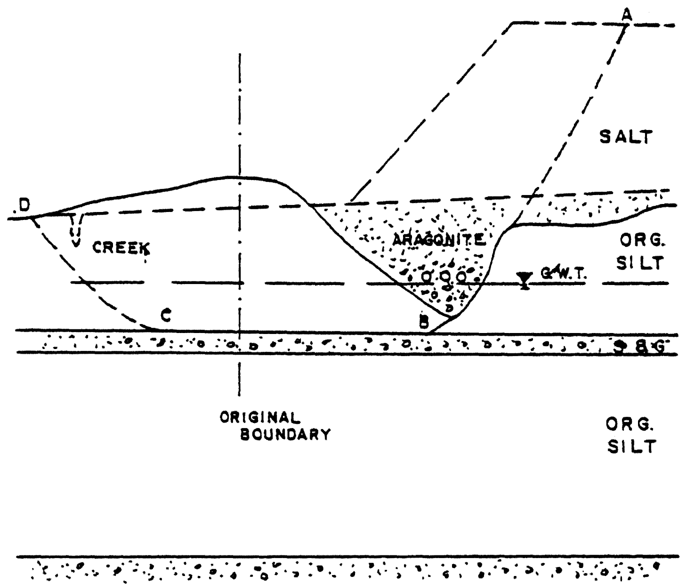

Fig. 5 Probable Mode of Failure

Computations for rotation-translational failure analysis are rather involved. The shear strength of the organic silt was taken, based on the test results provided, to be c = 300 psf (14.4 kN/m^2), which was the weighted average value for the upper organic silt layer without considering strength gain due to consolidation of the soil. The resulting factor of safety with respect to rotation-translational failure was computed to be 0.81 for the fill height of 30 ft (9.1 m) and 0.93 for 20 ft (6.1 m). Since the actual height at the time of failure was extimated to be approximately 28 ft (8.5 m), the factor of safety might have been about 0.84.

The use of the aragonite blanket and the wearing course had no doubt helped as they dissipated the applied load, especially wheel loads from trucks, and as the pad, as a whole, was subjected to bending in a horizontal plane. The high beam strength in the pavement and base strengthened the pad against plastic deformation. In view of this, the factors of safety calculated might have been higher than 0.84. However, there is no escaping from the fact that the shear strength of the supporting soil was not adequate to resist the combined translation-rotational mode of shear failure.

Further analysis shows that the maximum height of loading should have been limited (without considering the effect of the pavement) to 19 ft (5.8 m) for a factor of safety of 1.0 and to a height of 15 ft (4.6 m) for a safety factor of 1.3. This is illustrated on Fig. 4 as broken lines to indicate the maximum allowable load for given factors of safety and to illustrate an overloaded nature associated with the original design.

Stability Analysis With Strength Gain from Consolidation

Since organic soils increase in shear strength as they consolidate or as excess porewater pressures decrease, controlled rate of loading is beneficial and essential in loading of such soils. Using the coefficient of consolidation value provided of 21.8×10^{-4} cm^2/sec for the upper layer and employing the simplest analysis formula (Eq. 1) the gain in shear strength is calculated to be

$$\Delta c \text{ (tsf)} = 0.35 \ (\gamma H)U = 0.293 \ U \qquad (2)$$

where γ = unit weight of salt, H = loaded height and U = degree of consolidation.

Based on the above, the increase in cohesion at the time of loading completion (t = 7.5 days) was calculated to be 65 psf (3.1 kN/m^2). With this value the maximum safe height of fill for a factor of safety of 1.3 was to have been 17 ft (5.2 m), 60 cm more than the initial estimation, but far lower than the 28 ft (8.5 m) actually used.

Using the gain in shear strength due to consolidation which progresses as a function of time, an analysis was made to show what should have been the safe height of fill (for a safety factor of 1.3) as shown in Fig. 6. The figure clearly depicts the overloaded nature of the storage pad and its inevitable failure due to insufficient bearing capacity of the underlying organic soil.

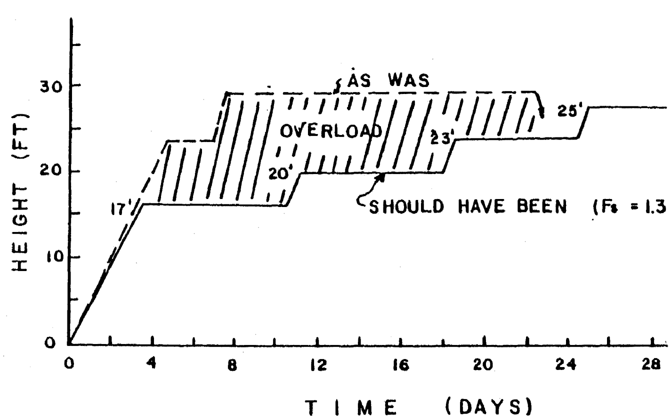

Fig. 6 Safe Heights of Stockpile

Settlement Calculations

Settlement of a cohesive soil is due to both ir mediate (elastic) deformation and consolidatioı Based on the stress change, the modulus of ela: ticity and Poisson's ratio, the immediate settlement was calculated to be approximately 3.0 ft (91 cm) at the center and 1.5 ft (4.5 cı at the corner of the loaded area, taking into account the effect of the pavement on the stress distribution and corresponding settlement. Since the elastic deformation is supposed to have occurred as soon as the loading was completed (after 7 days), the rapid nature of this loading had no doubt faciliated the process of failure because insufficient time was allowed to dissipate the excess porewater pressure which had built up upon loading. The magnitude of ultimate settlement due to consolidation was estimated to be 2.2 ft (67 cm). In highly organic deposits the secondary compression effects are likely to dominate, and for this particular project, the compression-time curves were characterized by a typical Type II consolidation in that the secondary compression began upon loading.

It was also estimated that it would have taken approximately a year to have 50% consolidation or one-half of the ultimate settlement. At th‹ time of loading completion (7.5 days), the degree of consolidation was estimated to have been about 15%, or a consolidation settlement of about 0.33 ft (10 cm). This means that the actual settlement that occurred prior to failure was due almost entirely to elastic deformation.

Findings

The Salt Storage Pad failed in a combined translation-rotational mode in shear resulting from the inadequate shear strength of the undeı lying organic soil which was loaded over a short period of time with high increments of loading, so that there was not a sufficient amount of time for the soil to consolidate and gain additional strength. This resulted in large vertical settlements and substantial horizontal movement of the soil mass.

It is glaringly evident that in the original design of the storage pad, the subsurface exploration and testing program were inadequate which resulted in the poor design. Furthermore the shear failure at the site could have been avoided if the concept of preloading were utilized in which the gain in shear strength oi the soil under slow incremental or stage loading (as shown in Fig. 6) was taken advantage oi An alternative was was to have counterberms placed at the toe of the fill, thereby minimizing the lateral and rotational movement of the soil mass. In either case, it was essential tc instrument the site to monitor the performance of the fill during and after construction in aı attempt to control the rate of loading compatible with the movement and strength change in the underlying soil.

REMEDIAL DESIGN

It is appropriate to mention briefly the remedial design that was proposed to the owner. After the failure, the salt was removed and a series of measurements were made to obtain the exact profile of the failure plane. Additional borings were also taken to determine the change in the properties of the soil. The subsided area was later backfilled with aragonite to the original ground surface level as shown in Fig.7, and drains were installed in the aragonite at the groundwater level.

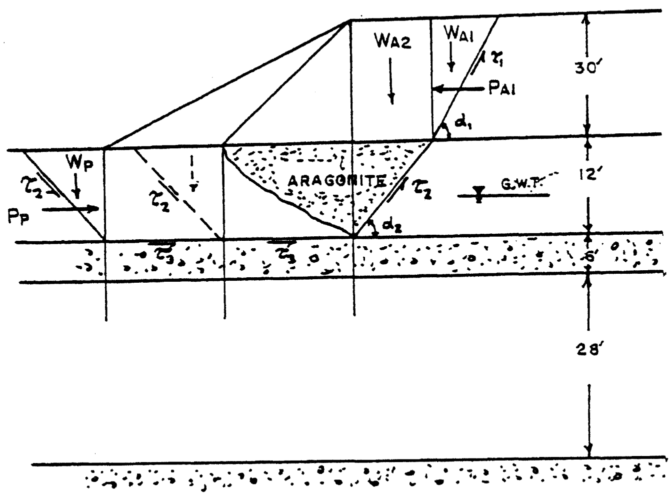

Fig. 7 Stability Analysis (Remedial Design)

Stability Analysis and Proposed Loading Scheme

In Fig. 7 are shown potential shear failure planes under the present (postfailure) condition. The possible and probable plane of failure is likely to be along the interface between the aragonite and the organic silt on the active side and through the disturbed organic silt layer on the passive side. A stability analysis was made of the proposed storage pad with a shear strength value of 340 psf (16.3 kN/m^2) and a factor of safety of 1.3. Two schemes were considered: one with a counterberm at the toe and the other without. It was also recommended to install a minimum amount of field instrumentation (see below) to monitor the performance of the fill during and after the construction.

Field Instrumentation and Monitoring

Uncertainties concerning the behavior of soils during and after construction are usually taken into consideration by using conservative design procedures or construction methods that increase the cost of the work while not necessarily assuring the desired safe performance. In most cases, this type of work can be realistically approached by installing measuring instruments, closely observing the behavior of the foundations, and altering the construction procedures if the measurements indicate that this might be necessary.

The extent to which the field control measurements are provided depends on the degree of sophistication one wishes to obtain and what expenditure one can afford for the purpose of controlling the field performance. Field instrumentation and monitoring for the present sites (salt and aragonite stockpiles) included a system consisting of two settlement gages or platforms, three piezometers and two inclinometer casings. For the aragonite storage an additional set of settlement gage, piezometer and inclinometer casing was provided on the southern boundary.

SUMMARY AND CONCLUSION

In the construction of any embankment or fill, the most important and critical factors are the rate of loading and the shear strength of the underlying soil. It is essential to obtain as much geologic and soil information as possible and to ascertain the nature and extent of the subsoils, especially when weak and organic soils are encountered. The soil exploration program at the site was not extensive. The laboratory test program was meager and the results were inconsistent. It was difficult to determine, for example, the change in degree of consolidation and corresponding change in the shear strength of the soil.

At this site one has very difficult soils to work with, namely the organic silt and clay. Because of the long-term stability characteristics (creep and plastic deformation), theoretical analysis and laboratory determination of their properties often defy the actual behavior of the soils and the foundations resting upon them. Furthermore, one is dealing with a very large structure (embankment) extending over a large area. For these reasons, field instrumentation and performance monitoring are a "must", not only to prevent failures but also to provide less conservative design and to run an efficient operation of storage activities.

The case of the failed Salt Storage Pad well amplifies the points of weakness just described. It was a simple case of shear failure resulting from inadequate shear strength of the underlying soil, which was loaded over a very short period of time. Therefore, sufficient time was not allowed for the soil to gain strength from the consolidation process.

Even when the rate of loading is controlled to the point of not inducing shear failure, plastic clay exhibits a creep phenomenon and plastic deformation, so that the performance of the soil and foundations must be overseered over a long period of time; or, in the anticipation of its long-term stability characteristics, the strength values used in the initial design must be considerably reduced.

REFERENCES

CHAE, Y.S., (1978), "Soil Engineering Problems and Review of Aragonite and Salt Storages, Ploughshare Point, New Jersey," Technical Report.